On The Uncertainties Related To The Estimation Of Extreme Environmental Condition

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ON THE UNCERTAINTIES RELATED TO THE ESTIMATION OF EXTREME ENVIRONMENTAL CONDITIONS

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by

H.F. Burcharth

INTRODUCTION

The calculation of the forces on the structural members of a structure in the sea is based on knowledge of the kinematics of the surrounding water and air. Therefore our goal is to establish some statistics for the related *velocity and acceleration fields.*

If we consider the sea such statistics can be established only with the knowledge of the joint probability density function (j.p.d.f.) related to *waves*, *current* and *sea level.*

Because waves and currents are described in terms of characteristic parameters as e.g. the significant wave height, H_s or H_{mo}, the mean zero-crossing period, T_z, the mean current velocity over the depth etc. we also need information on *parameter distributions* and *theories for particle kinematics* to make possible the calculation of the related velocity and acceleration fields.

With such information in hand one can in principle calculate the forces on a structural number and establish the j.p.d.f. of the force (moment, shear) and its direction, which again allow the estimation of extreme design values corresponding to some exceedence probability levels.

Besides this we need an evaluation of the uncertainty related to the extreme values.

The j.p.d.f. of waves, current and sea level can only be found with reasonable accuracy either by *direct measurements* on the location on question or by the use of *numerical models* based on meteorological data.

Due to lack of data the author is not able to evaluate the uncertainty related to extreme estimates on the combined wave-current-sea level effect.

In the following discussion of uncertainty only one parameter is dealt with, namely the wave height, which of course is a key parameter. Moreover, only the uncertainty related to estimates on extreme design waves is discussed. This means that uncertainty on long-term wave loads leading to fatigue failures is not included.

SOURCES OF ERROR ON WAVE ESTIMATES

The procedure for establishing the design data is shown in Figure 1. It also renders the many sources of errors. The figure indicates that although we can estimate a design wave in terms of height, period and direction we are still lacking information, physical understanding and theory for the estimation of the related velocity and acceleration fields. For this reason it is of course impossible to evaluate the uncertainties on this very important part of the procedure.

If we look only on the first part of the procedure which leads to the estimation of some extreme value of the significant wave height H_s the sources of errors can be listed as follows:

Data Collection

Measurements
Visual observations
Hindcast

$\downarrow$ Statistical treatment of data

Extreme Sea State

Characteristic wave height, e.g. H_s or H_{mo}
Characteristic wave period, e.g. T_z
Correlation of H and T
Wave direction
Persistance
Wave height distribution

$\downarrow$

Design Waves

Wave height, H
Wave period range
Wave direction range

$\downarrow$ (Information and physical understanding lacking)

Detailed Description of Design Wave(s)

3-dimensional geometry including
Height, crest level, asymmetries
Wave period
Wave direction

$\downarrow$ (Wave theory lacking)

Design Velocity and Acceleration Fields

Figure 1. *Procedure for establishment of extreme design wave kinematics.*

A. Errors in measurements, visual observations or hindcasting of the wave data on which the extreme statistics are based.

B. Errors related to extrapolation from short samples to events of high return periods, i.e. low probability of occurrence.
Errors due to the choice of exceedence level.
Errors due to the method of fitting data to a chosen distribution.

C. Lack of knowledge about the underlying distribution for the extreme events.

D. Errors due to plotting positions.

E. Climatological variations.

ad A. Errors in wave data

Le Mehaute et al., 1984, discussed the uncertainties and systematic errors or bias related to the wave data under the assumption of errors being normally distributed. They reported the following "typical" normalized standard deviation σ' defined as the absolute standard variation divided by the expectation ("mean") value of H_s:

Direct wave measurement	$\sigma'_M = 0.05$	bias 0.00
Visual observations from ships	$\sigma'_M = 0.20$	bias 0.05
Hindcast (excluding hurricanes and other tropical storms)	$\sigma'_M = 0.15$	bias 0.05

It should be noted that the two last set of figures are applicable only when the sample populations are ranked statistically. A direct comparison in the time domain, i.e. comparison of individual sea states, generally shows larger discrepancies. Moreover the figures are average figures. For instance it is believed that wave data based on to-day's most advanced hindcast models applied to relatively restricted areas, such as the North Sea where high quality weather maps are available will show a smaller uncertainty.

Based on comparisons of hindcasts with measurements by Resio and Vincent, 1978; Holthuijsen, 1980; Ewing et al., 1981; Bouws et al., 1982, the following generalized conclusions were stated by Battjes, 1984:

" (a) The development of new wave models or modification of existing ones, in response to the advances made in understanding of the physics of wave generation and propagation, has not (yet) given rise to a correspondingly better performance of these models operatinally. At least two factors contribute to this paradoxical situation. One is the limited quality of the wind input. Another is the fact that models with less realistic modelling of the physics compensate for this with more empiricism. It is not surprising that such models, tuned so as to simulate not only observed growth rates but a condition of saturation as well, are cabable of fairly realistic predictions in the more or less common situations similar to those for which they were tuned. It is to be expected however that the physically better founded models will perform better in more demanding conditions.

(b) For the majority of the models, the overall r.m.s. relative difference between hindcast and observed H_s is in the range 0.2 to 0.3, but the error in the prediction of extreme conditions is smaller by approximately a factor 2 (apart from possible phase

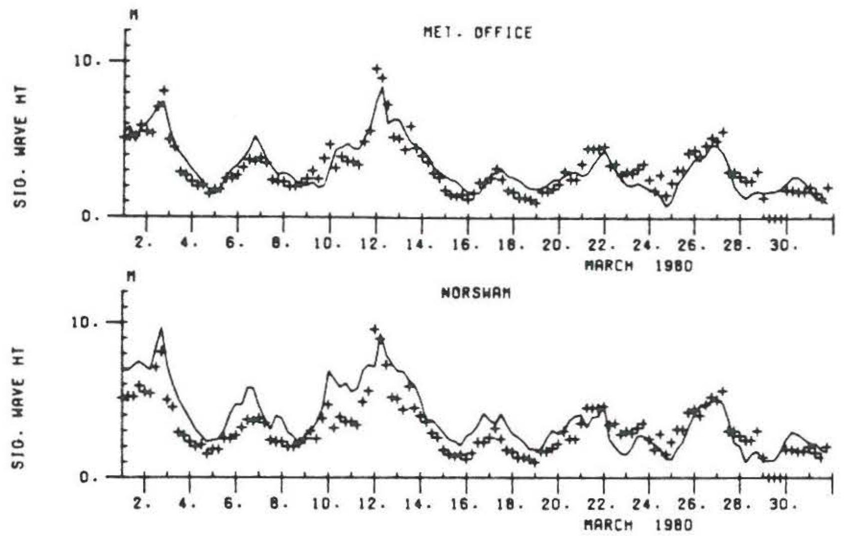

Fig. 4. Measured (+) and hindcast (——) significant wave height at Brent B (Ewing et al., 1981).

errors). An example is shown in Fig. 4 where results of a hybrid model (NORSWAM) and of a discrete model with nonlinear coupling (METOFFICE) are compared to observed H_s-values at the Brent B location. "

ad B. Errors due to short samples.

Estimates on events of low probabilities are often performed in the following two different ways:

1) The extrapolation of data from frequent measurements or observations. The data are often compiled at intervals $\Delta t = 3, 6$ or 24 hours, which gives a large sample, N events, even in the case of a short time of observation or record length Y in years. The order of magnitude of N is often 1000 - 10,000.

2) The extrapolation of relatively few data sets representing the max significant wave height H_s for a number of storms exceeding a certain level, H_s'. The data are often determined from hindcasts and the sample size N is typically within the range 10 - 30.

Wang et al., 1983, examined the uncertainties related to the first method. They considered the long term distribution of H_s to be of the exponential type which also includes the often used Weibull distribution,

$$P(H_s) \equiv P[H \leqslant H_s] = 1 - \exp(-(\frac{H_s - A}{B})^{\gamma}) \tag{1}$$

where A is signifying the background noise level or lower-bound, B is the scale parameter and γ is the shape parameter. All three characteristic variables are normally determined by best fitting to the observed data.

Assuming the data asymptotically normally distributed around the underlying probability distribution function, eq (1), the authors obtained for large N the normalized standard deviation,

$$\sigma_s' = \frac{1}{\gamma \ln(R\,\nu)} (\frac{R}{Y})^{0.5} \tag{2}$$

where R is the return period in years, ν is the number of observations per year compiled at interval Δt and Y is the number of years of observations. Formula (2) is valid only for low probability levels and only for large samples $N = \nu Y$ of uncorrelated data. The latter implies that Δt should exceed approximately 24 hours, but because of little sensitivity on the confidence bands for H_s smaller values, as for example $\Delta t = 6$ hours, are often used.

It is stressed that the data to be used must belong to the same statistical population as the extreme event in question, i.e. wave data must be separated with respect to origin, direction, shoaling effects etc. when relevant.

Example.

Taking $R = 50$ years, $Y = 5$ years, $\nu = 365$ observations per year and $\gamma = 1.2$ gives $\sigma_s' = 0.27$

Changing R and Y to 100 years and 3 years respectively gives $\sigma_s' = 0.46$

The second method mentioned above is relevant to situations where data have to be obtained from hindcasting, which, due to the costs involved, restricts the number of data.

Rosbjerg, 1981, considered this case, where only maximum values η of H_s for independent storms exceeding a chosen level H_s' are taken into consideration, cf. figure 2.

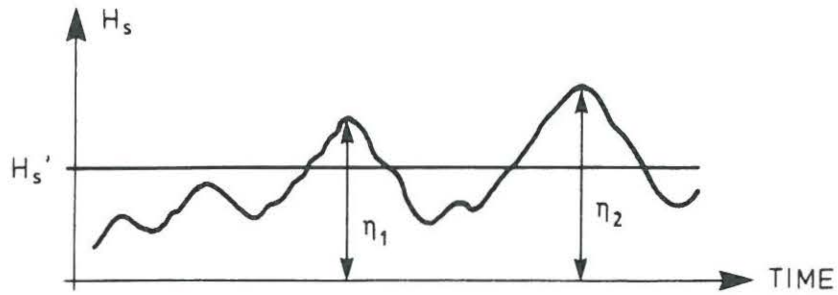

Figure 2. Data reduction by application of exceedence level, H_s'.

Rosbjerg assumed all the exceedences $\eta - H_s'$ to follow the exponential probability distribution,

$$P(H_s) \equiv P[\eta \leqslant H_s] = 1 - \exp(-\frac{H_s - H_s'}{\alpha}) \qquad (3)$$

which is of the same type as the Weibull distribution, eq (1), with $\gamma = 1$.

The author also assumed the events η to occur at times corresponding to a Poisson-process with time dependent intensity. He arrived at the following expression for the R-year event defined as the value of η, which in average is exceeded once every R years,

$$H_s = H_s' + \alpha \ln \nu R \qquad (4)$$

The corresponding absolute standard variation is

$$\sigma_s = \frac{\alpha}{(\nu Y)^{0.5}} (1 + (\ln \nu R)^2)^{0.5} \qquad (5)$$

and the normalized standard deviation consequently

$$\sigma_s' = \frac{\sigma_s}{H_s} = \frac{\frac{\alpha}{(\nu Y)^{0.5}} (1 + (\ln \nu R)^2)^{0.5}}{H_s' + \alpha \ln \nu R} \qquad (6)$$

The maximum likelihood estimate for α is

$$\hat{\alpha} = \bar{\eta} - H'_s \tag{7}$$

where $\bar{\eta}$ means average of η.

Nielsen et al., 1985, extended the analysis to include the Weibull distribution

$$P(H_s) = P[\eta \leqslant H_s] = 1 - \exp(-(\frac{H_s - H'_s}{\alpha})^\gamma) \tag{8}$$

and found the following

$$H_s = H'_s + \alpha(\ln \nu R)^{1/\gamma} \tag{9}$$

$$\sigma_s = (\ln \nu R)^{1/\gamma - 1}\left[\frac{\alpha^2}{\gamma^2 \nu Y} + (\ln \nu R)^2 \frac{\alpha^2}{\nu Y}\left(\frac{\Gamma(1+\frac{2}{\gamma})}{\Gamma^2(1+\frac{1}{\gamma})} - 1\right) + \frac{\alpha^2}{\gamma^4}(\ln \nu R)\cdot \ln(\ln \nu R))^2 \operatorname{Var}[\hat{\gamma}]\right]^{0.5} \tag{10}$$

ν is the average number of data per year and Γ the Gamma function.

The variance of $\hat{\gamma}$, $\operatorname{Var}[\hat{\gamma}]$, cannot easily be estimated, but by means of numerical simulation it is found that the term in (10) containing this quantity is highly dependent on the method for estimating the parameters in the Weibull distribution.

Petrauskas and Aagaard, 1971, found, by using a least square method, that the last term in (10) is insignificant. In this case the normalized standard deviation is

$$\sigma'_s = \frac{\sigma_s}{H_s} \cong \frac{(\ln \nu R)^{\frac{1}{\gamma}-1}\left[\frac{\alpha^2}{\gamma^2 \nu Y} + (\ln \nu R)^2 \frac{\alpha^2}{\nu Y}\left(\frac{\Gamma(1+\frac{2}{\gamma})}{\Gamma^2(1+\frac{1}{\gamma})} - 1\right)\right]}{H'_s + \alpha(\ln \nu R)^{1/\gamma}} \tag{11}$$

Nielsen et al., 1985, fitted the Weibull parameters by the method of moments, i.e. equating the first three moments of the distribution to those of the data, and found that the last term in (10) was of significance, namely in the order of 1/3 of the total standard deviation. The estimates on the parameter by the applied method of moments are given by

$$\frac{\Gamma(1+\frac{3}{\hat{\gamma}}) - 3\Gamma(1+\frac{2}{\hat{\gamma}})\,\Gamma(1+\frac{1}{\hat{\gamma}}) + 2\Gamma^3(1+\frac{1}{\hat{\gamma}})}{(\Gamma(1+\frac{2}{\hat{\gamma}}) - \Gamma^2(1+\frac{1}{\hat{\gamma}}))^{3/2}} = \frac{\overline{\eta^3} - 3\,\overline{\eta^2}\,\bar{\eta} + 2(\bar{\eta})^3}{(\overline{\eta^2} - (\bar{\eta})^2)^{3/2}} \tag{12}$$

$$\hat{\alpha}^2 = \frac{\overline{\eta^2} - (\bar{\eta})^2}{\Gamma(1+\frac{2}{\hat{\gamma}}) - \Gamma^2(1+\frac{1}{\hat{\gamma}})} \tag{13}$$

$$\hat{H}'_s = \bar{\eta} - \hat{\alpha}\,\Gamma(1 - \frac{1}{\hat{\gamma}}) \tag{14}$$

$\overline{\eta^2}$ and $\overline{\eta^3}$ mean the average of sample values of η^2 and η^3, respectively, which are unbiased estimates of $E[\eta^2]$ and $E[\eta^3]$.

Eqs (1), (6) and (11) make it possible to determine the necessary sample length when a prediction for a given return period with a prescribed accuracy and confidence is required. If we assume that the dispersion of the sample variables is normal distributed around the underlying probability distribution function then it is possible to determine the control curves corresponding to various confidence limits, see e.g. Wang et al., 1983. The limits 0.84 σ_s', 1.28 σ_s', 1.65 σ_s' and 2.32 σ_s' define the *upper* bound of spread corresponding to a confidence level of 80%, 90%, 95% and 99% respectively. For instance, the prediction of an event with 90% confidence (i.e. 90% of the observations should fall below the upper bound control curve) and an uncertainty of no more than 0.20 implies that 1.28 $\sigma_s' \leqslant 0.20$. Inserting this in eqs (1), (6) and (11) gives the corresponding number of years of observation Y for given ν and R.

Example.

The accuracy of estimates based on a restricted number of hindcasted data sets might be illustrated by the following example. The Delft Hydraulics Laboratory made a hindcast study for a specific deep water location in the Mediterranean Sea and found for a 20 years period the following 17 most severe storms, Table 1:

Table 1. Example of hindcasted storm wave data for a 20 years' period.

Rank i	Max H_s (= η) metres	Peak period T_p seconds	Average wave direction degrees
1	9.32	14.0	143
2	8.11	14.1	139
3	7.19	13.4	123
4	7.06	10.8	123
5	6.37	11.9	143
6	6.15	11.1	185
7	6.03	12.3	135
8	5.72	10.5	176
9	4.92	10.7	150
10	4.90	10.6	129
11	4.78	11.8	161
12	4.67	9.9	120
13	4.64	9.2	122
14	4.19	10.5	137
15	3.06	11.1	154
16	2.73	8.2	153
17	2.33	8.3	126

If we choose H_s' = 4.0 m we find N = 14 storms exceeding this level over a period Y = 20 years, which gives ν = 14/20. According to eq (7) α can be estimated to $\hat{\alpha}$ = 2.00 m. It can now be tested if the data follow the assessed distribution, for example the exponential type given by eq (3). In this case a straight line with slope 1:1 should be obtained by plotting $\eta_i - H_s'$ against $-\hat{\alpha}\ln(1 - P(\hat{\eta}_i))$, where $P(\hat{\eta}_i) = 1 - \frac{i}{N+1}$, (Gumbel plotting positions). Figure 3 shows that the fit is reasonable.

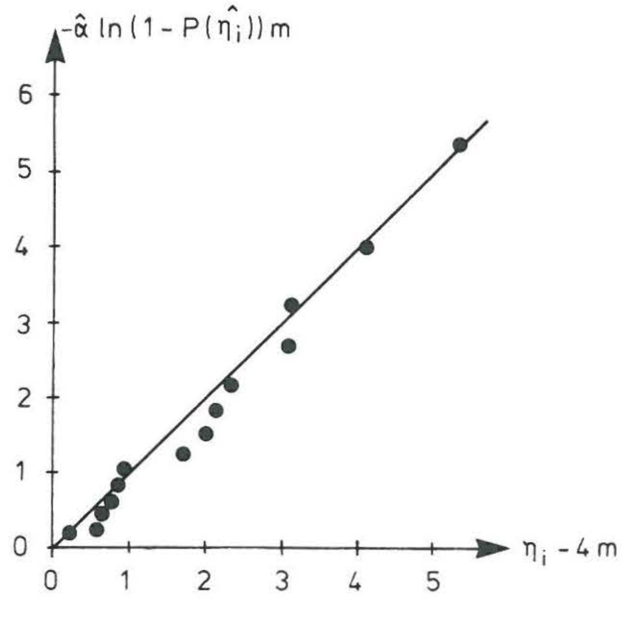

Figure 3. Test on exponential distribution of wave height exceedences.

Formulae (4) - (6) are then valid and the expectation values and the standard deviations can be calculated for various return periods, for instance

Return period R years	H_s metres	σ_s metres	σ'_s
50	11.11	1.97	0.18
100	12.50	2.33	0.19

Note that a change in the exceedence level H'_s for example to 3.50 m, which still gives N = 14, will change H_s and σ_s significantly since for R = 50 years H_s = 12.39 m, σ_s = 2.47 m, σ' = 0.20 and for R = 100 years H_s = 14.12 m, σ_s = 2.92 m, σ' = 0.21 m. This important problem is not discussed further here.

It is obvious that the 14 data points also fit a Weibull distribution.

If all the 17 data points given in Table 1 are considered, it corresponds to a exceedence level of $H'_s \cong 2.25$ m because the lowest value in the data set is H_s = 2.33 m. It turns out that in this case the data do not fit neither the exponential distribution, eq (13), nor the Weibull distribution, eq (8). However, if the exceedence level is not interpreted as the physically true cut-off level, but is regarded a fitting coefficient only, like α and γ, then the 17 data points follow the Weibull distribution very closely, as demonstrated in Figure 4. The coefficients are in this case H'_s = 0.73 m, α = 5.27 m and γ = 2.80, all estimated by the method of moments.

From eqs (9) - (11) we obtain the following corresponding values

Return period R years	H_s metres	σ_s metres	σ'_s
50	9.19	0.88	0.10
100	9.71	0.97	0.10

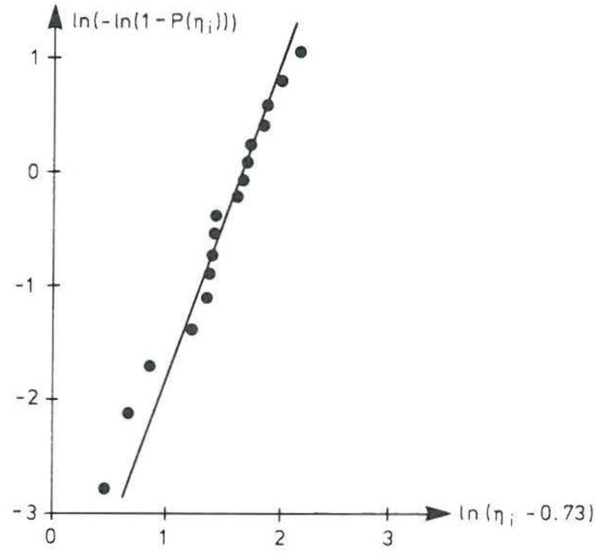

Figure 4. Data fit to the Weibull distribution. Gumbel plotting positions.

The Weibull distribution shown by the straight line in Figure 4 is a result of the chosen method of fitting. A least square fit or a visual fit will produce different lines and different estimates on the extreme events.

Thus it is concluded that also the choise of exceedence level and the method of fitting the data to a chosen distribution introduce uncertainty on the estimates of extremes.

As discussed previously the confidence limits for an extreme estimate can be determined under certain assumptions. Figure 5 illustrates this by showing an example of the variation of the 80% probability control curves corresponding to various return periods for the data in Table 1 of which the 14 largest storms follow the exponential probability distribution, eq (3), cf. Fig. 3.

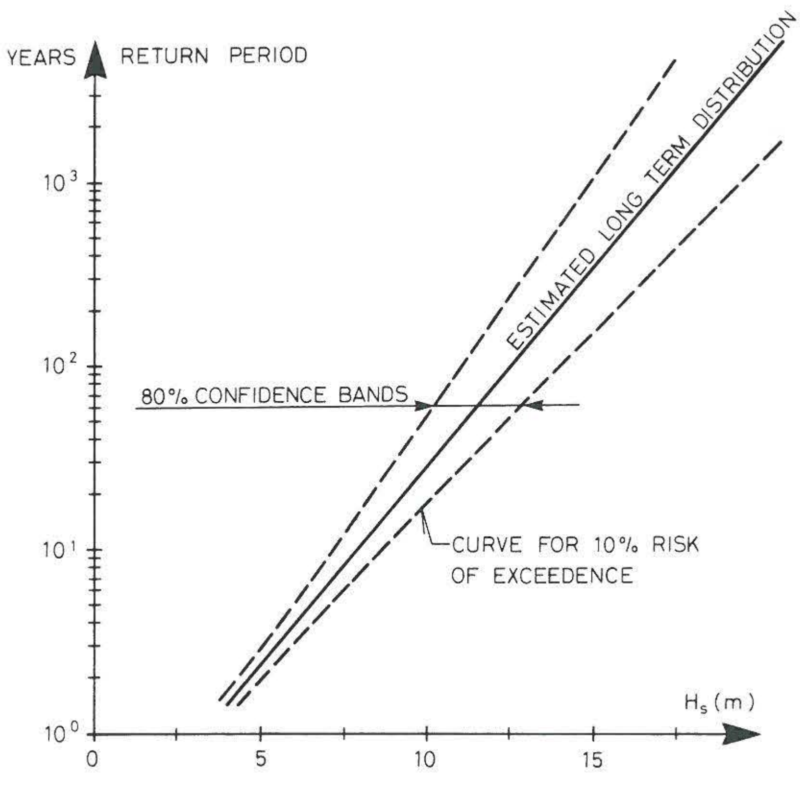

Figure 5. *Example of 80% probability control curves for a data set of 14 storms in a 20 years' period.*

The figure clearly shows the positive influence of a longer period of observation.

When evaluating the *R-year event* (the return period R), it is very important to notice that this event has a probability E of being equalled or exceeded (encounter probability) in the specific lifetime L of the structure.

The relationship is

$$E = 1 - (1 - \frac{1}{R})^L \tag{15}$$

which in the case of large R can be approximated

$$R = - \frac{L}{\ln(1 - E)}$$

The encounter probability is illustrated in Figure 6, which shows that for a specific long term storm wave history a structure with a 20 years' lifetime might experience very different wave loads dependent on the actual location in time.

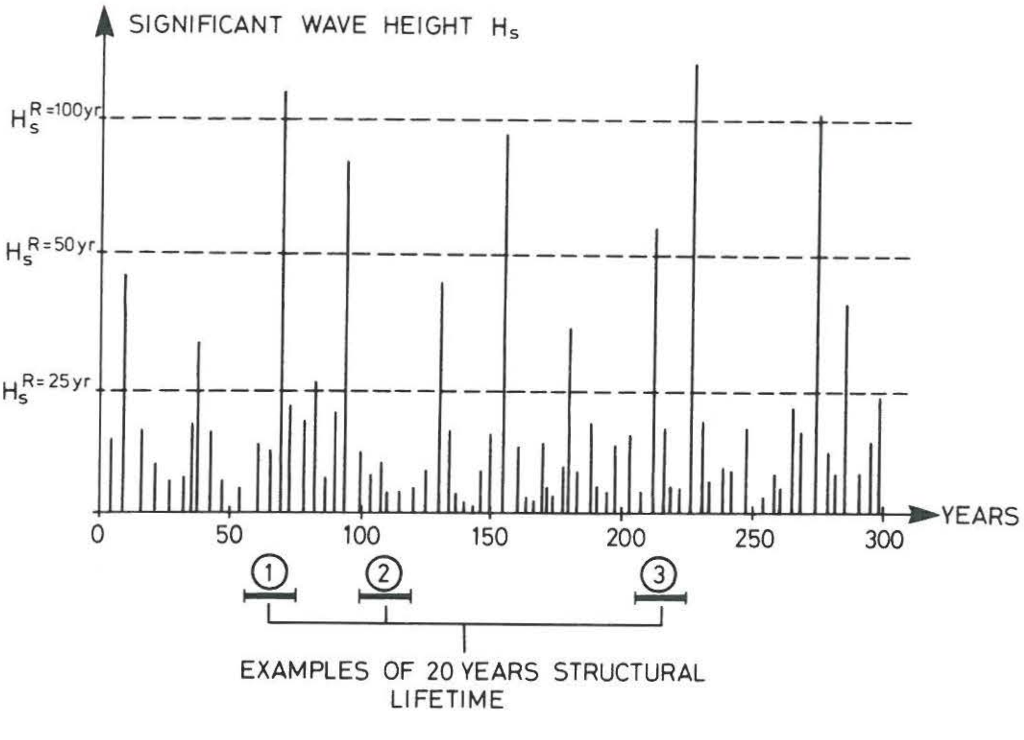

Figure 6. Illustration of encounter probability.

If for example the structural lifetime is 20 years and a 10% exceedence probability is acceptable the structure must be designed for the 190 years' event, R = 190 years. If the lifetime is increased to 50 years the design level rises correspondingly to R = 500 years.

Many offshore structures with an estimated lifetime of 20 years are designed for the 50 years' event which implies an exceedence probability as high as 33%. Using as an example from Figure 5 the upper bound 80% probability curves means a design H_s of 12.5 m. But if for the same structure an encounter probability of 10% corresponding to R = 190 years is applied then the design H_s increases to 15.4 m.

ad C. and D. Errors due to the lack of knowledge on the true long term distribution and due to plotting positions.

Several probability distributions are used to describe extreme wave height statistics. These include for example the log-normal distribution, the extremal type I or Gumbel or Fisher-Tippett I distribution, the extremal type II or Fretchet or Fisher-Tippett II distribution, the Ward-Borgman distribution and the extremal type III or Weibull distribution. Although each of these distributions has a theoretical base, they cannot be evaluated and related to the extreme waves on a physical base. As a consequence they are only fits to the available data. Most often the scales used for the plotting are such that the chosen distribution lies on a straight line, simply because of the more convenient visualization of the extrapolation. However, when extrapolating, one should always be aware of possible physical processes, such as for example wave breaking, which might interrupt the probability distribution at some probability level.

It follows from these comments that due to unknown extreme distribution errors can only be estimated by a sensitivity analysis in which various distributions are fitted. Table 2 shows such an analysis by the Delft Hydraulics Laboratory performed on the wave data given in Table 1.

Table 2. Example of influence of choise of extremal distribution and plotting position on low-probability wave heights. Data by Delft Hydraulics Laboratory.

Extremal distribution	Plotting position	Correlation coefficient	Return period H_s 50 year	100 year
Type I Gumbel	Gumbel	0.9875	11.0 m	12.2 m
	Gringorten	0.9852	10.3 m	11.3 m
Ward/Borgman	Gumbel	0.9872	9.8 m	10.5 m
	Gringorten	0.9920	9.4 m	10.1 m
Type III Weibull	Gumbel	0.9877	9.6 m	10.2 m
	Gringorten	0.9877	9.3 m	9.9 m

Although no accurate figures can be given it seems reasonable from this table and the above given example based on the distribution, eq (3), that due to unknown extreme distribution a normalized standard deviation σ'_D might be in the order of

$$\sigma'_D \cong 0.05 - 0.10.$$

for return periods of approximately 50 to 100 years, progressively increasing as the return period increases.

In order to plot the data a position formula must be adopted. Many different plotting positions, all based on some statistical considerations, exist, but it is not easy or possible to select a specific one as the most correct. For this reason it is reasonable to estimate the error due to plotting positions by sensitivity analyses involving a number of reasonable plotting rules.

Table 2 gives an example where only two plotting rules are used, namely

Gumbel/Weibull
$$P(\eta_i) = 1 - \frac{i}{N+1} \qquad (16)$$

and

Gringorten
$$P(\eta_i) = 1 - \frac{i - 0.44}{N + 0.12} \qquad (17)$$

It is seen that significant deviations in the estimated extreme wave height occur due to the plotting rules. It is believed that a realistic normalized standard deviation σ'_p on extreme events will be in the order of

$$\sigma'_p \cong 0.05$$

ad E. Errors due to climatological variations.

An additional source of uncertainty is the natural variation of the wave climate. Le Mehaute et al., 1984, considered this difficult problem under the assumption of the natural climatology being ergotic and stationary and governed by the statistical law of Weibull distribution. By setting Y = R in eq (2) they found that the normalized standard deviation of climatological variations in R years at a particular location is given by

$$\sigma'_C = \frac{1}{\gamma \ln(R\nu)} \tag{18}$$

If for instance we estimate $\gamma \cong 1.2$ as proposed by the authors we find for $\nu = 365$ and R = 50 or 100 years $\sigma'_C \cong 0.08$.

Combined errors.

The above mentioned sources of uncertainty can be assumed mutually independent except for an unknown but probably weak correlation between the climatological variation and the data samples.

The total normalized standard deviation might then be estimated by

$$\sigma' \cong (\sigma'^2_M + \sigma'^2_s + \sigma'^2_D + \sigma'^2_p + \sigma'^2_C)^{0.5} \tag{19}$$

With reference to the foregoing discussion one can establish the following two examples:

Examples.

Direct wave height measurement. ν = 365 observations per year. Y = 5 years. R = 50 years.

$$\sigma' \cong (0.05^2 + 0.27^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.30$$

Hindcasted wave heights. 14 data sets over Y = 20 years. R = 50 years.

$$\sigma' \cong (0.15^2 + 0.18^2 + 0.07^2 + 0.05^2 + 0.08^2)^{0.5} = 0.26$$

From this it is seen that, even with what is generally regarded reasonable lengths of data sample and observation period, the uncertainty related to the 50 years' event is significant and in the order of $\sigma' \cong 0.25 - 0.30$. If we assume normally distributed random variables it means a 16% probability of the wave height being bigger than 1.25 - 1.30 times the estimated height.

The uncertainty increases significantly when the lengths of data sample and the period of observation are reduced to figures below those given above. The uncertainty also increases with the design return period which might very well be well over 50 years, cf. the foregoing discussion on encounter probability.

CONCLUDING REMARKS

As said in the introduction a statistics suitable for the estimation of extreme environmental conditions can be established only with the knowledge of the joint distribution of the key parameters related to waves, current, sea level and wind. However, even when dealing with *one* key parameter, as for example the wave height, the usual shortage of data leads to large uncertainties on estimates of extremes as explained above. When dealing with joint data which are considerably more difficult to obtain, one might expect even larger uncertainties.

To-day it is normal practice to design offshore structures for the simultaneous effect of a certain extreme value of each environmental parameter, i.e. the 50 years' wave, the 50 years' current, the 50 years' wind . . . This of course means that the structure is designed for a total load which has a probability of occurrence less than once in 50 years. Very often this is interpreted as over-design. However, this is not necessarily the case when one thinks of the large exceedence probability corresponding to a 50 years' return period and a 20 to 50 years' structural safety. How-it might well be that this "conservative" approach leads to a reasonable structural safety. However, the problem is that without knowing the joint distribution of the key parameter we cannot evaluate the safety level. In other words we have no idea about the design load exceedence probability.

Joint data on wind, waves, currents and sea level are difficult to obtain. The most promising approach is to apply numerical hindcast models. However, it should be mentioned that when dealing with multivariate analyses the concept of a return period value has no meaning as it cannot be given a relevant interpretation. Therefore a better way might be to calculate the overall forces in the structure or the stresses in the structure as a combined function of wind, waves, current and sea level and thus reduce the extreme analysis to a single parameter analysis for which the return period concept is applicable.

Two things should be mentioned in this connection. Firstly the fact that the calculation of forces and stresses introduces other sets of uncertainties on top of those related to the environmental parameters. Secondly the observation that the return period concept is not very expedient as it might give biased conceptions because of the strong non-linear relationship between the return period and the key parameter values.

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